State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. RAMIN RASHEDI
Division of Structure Design
Design Office 59-232

Attention: Mr. Gary Blakesley

Date: June 5, 2000

File: 11-SD-5-KP 49.33
11-0301U1

Los Penasquitos Creek (Widen)
Bridge No. 57-0511

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations - MS 5
Office of Structure Foundations

Subject: Foundation Recommendations

Introduction

The proposed widening of the Los Penasquitos Creek Bridge (Br. No. 57-0511) is part of planned Route 5/805 Freeway improvements for the San Diego area. A Request for Final Foundation Recommendations (dated October 22, 1998) for the subject bridge was submitted to the Office of Structure Foundations (OSF) by Mr. Ramin Rashedi. Site specific ARS, liquefaction potential, and methods of liquefaction mitigation were requested in the above memorandum. A list of preliminary column/pile loads and shaft diameters (at the bents) were provided to OSF by Mr. Rashedi (dated December 18, 1998). As the 5/805 and 5/56 project has progressed, further revisions of the above pile load and shaft diameter list was sent to OSF including Revision 1 (dated February 24, 1999), Revision 2 (dated April 9, 1999), and Revision 3 (dated May 11 and 26, 1999). Abutment pile diameters and axial service loads were provided by Mr. Rashedi (February 1, 2000) who also requested P-Y curves or COM624 soil profile information at the abutments and final P-Y curves for the bents. Mr. Gary Blakesley provided final bottom of footing/pile cutoff elevations for the proposed bridge (Caltrans facsimile copy, dated March 24, 2000). P-Y curves were also requested by Mr. Earl Seaberg on February 24, 1999. In the same memorandum it was mentioned that in order to mitigate the effects of potential liquefaction, large diameter cast-in-drilled-hole piles (CIDH) would be used for structures at the 5/805 Interchange (Seaberg, February 24, 1999). In preliminary evaluations of the As Built Log of Test Borings (LOTBs) performed by the Office of Geotechnical Earthquake Engineering (Jones and Abghari, February 10 and April 7, 1999), potentially liquefiable soils are estimated at approximately up to 12.19 to 15.24 m (40 to 50 ft) thick. Mr. Rashedi (DSD, facsimile copy, December 23, 1998) requested that the existing foundations for the subject bridge be investigated, as this structure was not included in the previously completed seismic retrofit program.

Subsurface information was obtained by OSF drilling 2 - 94 mm and 1 – 80 mm diameter mud rotary borings which also involved extensive coring. The 80 mm diameter boring was subsequently rebored to a larger diameter to allow placement and grouting of 76 mm PVC casing for later downhole geophysical logging. Results from the field studies will be shown on the LOTBs. In addition to the recent field work, the As Built LOTBs for the Bridge Across Los Penasquitos Channel (Br. No. 57-0511), Contract No. 11-022454, dated July 1964, contained additional site and subsurface information and will be included within the new contract plans.

Site Description

The existing abutments are dominantly founded in approach embankment fill material which ranges between approximately 12.2 m (40.0 ft) thick for the southern abutment (existing Abutment 1) and 9.0 m (29.5 ft) thick for the northern abutment (existing Abutment 4). Underlying native alluvium (Holocene and possible older Quaternary alluvium, undifferentiated) ranges from approximately 10.82 to 21.03 m (35.5 to 69 ft) thick. Isolated to the south near Abutment 1, intertongueing formational sands of the probable Eocene Torrey Sandstone (uncemented, soil-like, very dense sand) and formational claystone/siltstone of the probable Ardath Shale (intensely weathered, soil-like, clay/silt) occur rather shallow from below elevations ranging from -1.71 to –5.00 m (-5.6 to –16.4 ft). The undulatory top surface of the more rock-like Eocene Ardath Shale (into which permanent casing will be placed) was encountered from elevations ranging from –7.07 to –12.86 m (-23.2 to –42.2 ft).

Approach embankment fill material consists dominantly of very stiff to firm/medium dense to loose, lean clay, clayey sand, and elastic silt with scattered gravel-size soft siltstone/claystone/sandstone rock fragments interlayered with silty sand and sandy silt with intermittent scattered gravel and cobbles. Native material [mapped as Holocene alluvium and slope wash undifferentiated according to Kennedy (1975) and probably including some older alluvium at depth], can be divided into two units with the upper sediments consisting of dominantly loose to medium dense/firm to stiff, silty sand, sand, and sandy silt interbedded with sandy lean clay with intermittent traces of gravel, clayey sand, and rare elastic silt. Organic material and black sandy clay to sandy silt can be found intermittently underlying embankment. The upper alluvial unit ranges from approximately 8.84 to 16.15 m (29 to 53 ft) thick. The upper alluvial unit also contains some medium dense to dense, silty sand lenses [up to 3.05 m (10 ft) thick]. The underlying native alluvial unit [from 1.2 to 9.6 m (4 to 31.5 ft) thick] found below elevations ranging from approximately +0.21 to –8.23 m (+0.7 to -27 ft) consists of generally dense to very dense/very stiff, gravel/cobble lenses with sand matrix interbedded with sand with intermittent scattered gravel, sandy lean clay, and clayey sand. Generally the extremely hard, subrounded to subangular gravel/cobbles [up to 200 mm (8 in) diameter], composed dominantly of metavolcanic rock fragments, directly overlie bedrock or occur as sporadic lenses within the underlying alluvial unit. Much of the loose native material is considered potentially liquefiable and is being investigated by the Office of Geotechnical Earthquake Engineering (OGEE) for potential mitigation measures or adequacy of proposed mitigation measures. As mentioned earlier, final p-y (lateral resistance) curves are also being developed for use at proposed bridge support locations. As mentioned above in the area beneath Abutment 1, remnants of probable Torrey Sandstone (uncemented, soil-like, very dense sand) intertongue with probable Ardath Shale (intensely weathered, soil-like, clay/silt) ranging from an estimated 5.79 to 7.01 m (19 to 23 ft) thick. The underlying more rock-like portion of the Eocene Ardath Shale generally consists of interbedded very soft to moderately hard, mudstone, sandy claystone, siltstone, and minor well cemented calcareous sandstone. The formation is generally slightly to moderately weathered, unfractured to slightly fractured, often thinly bedded, and contains occasional concentrations of pelecypod debris. Some fractures have calcite infilling. The typical Ardath Shale in this area is partially underlain and interfingers with the Eocene Torrey Sandstone at Los Penasquitos Creek (especially towards the southeast near Abutment 1 right side sliver widen, Boring 99-3). The intertongueing sandstones representative of the Torrey Sandstone are composed of minor thin [0.3 to 0.6 m (1 to 2 ft) thick] soft to very hard calcite cemented fine to medium sandstone and thicker [3.66 to 7.92 m (12 to 26 ft) thick] formational sand tongues (uncemented, soil-like, very dense sand). The formational sand lenses are most common to the southeast of the proposed bridge widenings. The very soft to moderately soft upper formational mudstones/claystones/siltstones of the Ardath Shale [0 to 9.45 m (0 to 31 ft) thick], were considered to possess weak rock unconfined compressive strengths of approximately 1240 kPa (180 psi). Below this upper zone, generally soft to moderately hard mudstone/claystone/siltstone/ and cemented sandstone (ranging from weak to fairly strong rock) show unconfined compressive strengths ranging from at least 1517 to 2068 kPa (220 to 300 psi) and higher. In areas where the formational uncemented sands are present, weak and strong rock formulas do not apply for calculation of pile/soil skin friction. While performing cast-in-drilled-hole (CIDH) pile calculations, OSF conservatively treated these sands as soil with significant overburden. The deepest boring for the bridge, Boring 99-3 (some distance from Abutment 1 – right side sliver widen), penetrated 54.62 m (179.2 ft) below the surface [elevation –41.54 m (-136.3 ft)]. Downhole P-S logging (compression and shear wave) showed that the better quality formational mudstones/claystones/siltstones had shear wave velocities averaging 530 to 576 meters per second (1740 to 1890 fps) which appear to correlate with unconfined compressive strengths of at least 724 kPa (250 psi) and higher (borderline weak to strong rock). These higher shear wave velocities were found within Boring 99-3 (near Abutment 1-right side sliver widen) in the competent Ardath Shale siltstone/claystone tongues below approximate elevation –7.41 m (-24.3 ft). The LOTBs should be reviewed for more specific details.

Surface Water and Scour

Surface water was often stagnant within Los Penasquitos Creek with only minor flow observed during the field investigation. Following a wet period on March 2, 2000, the author observed more substantial surface flow, however, flows were not enough to cause apparent substantial scour.

The proposed bridge widenings will span Los Penasquitos Creek. Both existing Bents 2 and 3 and proposed Bents 2 and 3 (to support the proposed widenings) are and will be located within the stream channel. It appears to OSF that remaining nearby supports are protected by embankment levees with rock slope protection. Only minor erosion or scour was found near the east side of existing Bent 2 for the bridge during OSF field investigation and brief survey of the stream bottom in the area. The As Built Borings B-2 and B-5 (Br. No. 57-0511) show up to approximately 6.1 m (20 ft) of loose, silty sand with some clay binder intergrading with clayey sand and sandy silt below the channel surface. The cohesive interbeds are less likely to be affected by stream scour. According to the Preliminary Report (Wang, February 19, 1999) for the Los Penasquitos Creek Bridge (Br. No. 57-0511L), total pier scour in this area is estimated at up to 2.5 m (8.2 ft).

OSF feels that potential scour should have negligible effect on the existing driven concrete pile foundations for the bridge due to adequate pile length and interbedded somewhat cohesive soils. The large diameter piles proposed to support the widenings will be affected very little by scour due to their extreme length and anchorage within unscourable rock material. Axial support is gained entirely within unscourable rock material. However, potential scour should be considered with regards to erosion and possible loss of lateral support within shallow soils at proposed in-channel bents. Due to the length of the piles, OSF feels that potential loss of significant lateral support is unlikely.

For further information, refer to Preliminary Investigations and Hydraulics reports in this area.

Ground Water

Ground water was not recently measured at the Los Penasquitos Creek Bridge. However, measurements within Boring 99-5 (Bent 2 – Right Side Widen) for the nearby Rte. 5/805 Separation (Br. No. 57-0512) revealed static ground water at elevation +8.84 m (+29.0 ft) measured March 2, 2000 (shortly after rains). Boring 99-5 is on the south side of Los Penasquitos Creek on the west shoulder of Rte. 805 upstream from the subject bridge. In the area of Boring 99-1 (near Bent 8, left side – left bridge widen) for the nearby Sorrento Viaduct (Br. No. 57-513L), static ground water was measured at elevation +7.77 m (+25.5 ft) on January 11, 2000. Boring 99-1 is south of the subject bridge on the east side of Sorrento Valley Road. The bottom of Los Penasquitos Creek in the area ranges from an estimated (elevations based on one survey point and estimates) +7.01 to +7.62 m (+23 to +25 ft) elevations and can be flooded. The ground water level fluctuated approximately from 0.03 to 0.3 m (0.1 to 1 ft) during OSF's recent investigation.

The As Built LOTB for the Bridge Across Los Penasquitos Channel shows ground water was encountered from approximate elevations +3.26 to +4.33 m (+10.7 to +14.2 ft) based on the City of San Diego datum, which requires a +2.45 m (+8.05 ft) add (Schuh, Caltrans Memorandum, March 7, 2000 and facsimile copy, February 14, 2000) to adjust to the current metric elevations (NAVD 88) upon which the recent plans and boring program are based. The adjusted to metric As Built elevations would then show ground water was encountered at elevations +5.71 to +6.78 m (+18.7 to +22.2 ft) for the earlier foundation investigation, with measurements taken during April 1962. The As Built LOTB for the slightly upstream (Rte. 805) Los Penasquitos Creek Bridge (Br. No. 57-0779) reveals ground water was encountered from elevations ranging from +6.58 to +6.40 m (21.6 to 21.0

ft). Again correcting English to metric (NAVD88) elevations would show ground water encountered from elevations +7.17 to +6.99 m (+23.5 to +22.9 ft) using an add of +0.588 m (1.93 ft).

Seismicity

See the memorandum (dated February 10, 1999) concerning Preliminary Seismic Design Recommendations sent to Mr. Seaberg from Mr. Ron Jones and Mr. Abbas Abghari. Final Seismic Design Recommendations and Lateral Resistance, p-y Curves will be submitted by the OGEE.

As mentioned above (Jones and Abghari, February 10, 1999) the proposed "structure is located approximately 5 km from the Newport-Inglewood-Rose Canyon fault which has a maximum credible earthquake moment magnitude of M=7.0 and based on the Caltrans California Seismic Hazard Map (Mualchin, 1995), this structure is within the peak horizontal bedrock acceleration zone of 0.5 g."

As mentioned above pseudo-rock-like material [Vs ranging from 530 to 576 meters per second (1740 to 1890 fps)] occurs below approximate elevations –7.41 m (-24.3 ft) to –14.02 m (-46 ft) measured within Boring 99-3 (southeast of Abutment 1 – right side sliver widen). Higher shear wave velocities were measured within siltstone/claystone tongues up to approximately 5.2 m (17 ft) thick.

Liquefaction

Liquefaction potential is considered moderate to high. Holocene and older Quaternary alluvium (undifferentiated) at the site is dominantly composed of loose to medium dense/firm to stiff, silty sand, sand, and sandy silt interbedded with sandy lean clay, clayey sand, and rare elastic silt. Ground water is also rather shallow and is estimated to be present from approximately 3.35 to 4.27 m (11 to 14 ft) below the south stream bank and has been observed flowing within Los Penasquitos Creek during the recent field investigation. Preliminary analysis (Jones and Abghari, February 10 and April 7, 1999) estimates that the upper 12.19 to 15.24 m (40 to 50 ft) of soils are considered potentially liquefiable. As mentioned above, final liquefaction potential is being determined by the OGEE. OGEE (Mahallati and Abghari, May 18, 2000) has recently determined that potentially liquefiable deposits exist in the area of the bridge and Los Penasquitos Creek both on the north bank [Boring 99-1 (Abutment 4, Br. No. 57-511) is the same shared boring as Boring 99-18 (Bent 14, S5/S805 Truck Connector, Br. No. 57-1069F)] and also on the south bank.

As Built Foundations

OSF's review of the "As Built" Plans and LOTBs supplemented by recent additional borings with laboratory test results allow calculation of ultimate compression and tension capacities for the existing Class II driven concrete piles used at this bridge, which are shown below.

Support Location/ Pile Type	Average Corrected (NAVD 88) Pile Tip Elevation		Ultimate Pile Compressive Capacity		Ultimate Pile Tension Capacity	
	m	feet	kN	tons	kN	tons
Abutment 1/Class II	-3.05	-10.0	925	104*	418	47*
Bent 2/Class II	-3.35	-11.0	1094	123*	115	13*
Bent 3/Class II	-6.40	-21.0	1121	126*	80	9*
Abutment 4/Class II	-9.14	-30.0	1459	164*	516	58*

*Ultimate pile compression and tension capacities are calculated assuming liquefiable soils above approximate elevation +1.52 m (+5 ft) beneath Abut 1 and Bent 2. Beneath Bent 3 and Abutment 4, OSF assumes liquefiable soils occur above approximate elevation –5.49 m (-18 ft) and a nonliquefiable soil layer exists between approximate elevations +2.44 to –0.91 m (+8 to –3 ft).

Foundation Recommendations

The following recommendations are based on the Los Penasquitos Creek Bridge (Widen) General Plan (revised May 3, 1999), Foundation Plan (1 sheet, checked by S. Wang, October 14, 1998), the above mentioned memorandums and communications from Mr. Rashedi (Caltrans facsimile copy dated May 26, 1999 regarding pile loads and bent pile diameter and a memorandum supplying abutment pile diameters and service loads, February 1, 2000) and Mr. Gary Blakesley (Caltrans facsimile copy with final bottom of footing elevations, dated March 24, 2000).

Fills can be placed in accordance with Section 19-6 of the Standard Specifications. End dumping is not permitted. At the Abutment 1 area, any settlement due to the addition of the right side sliver fill widening should be negligible in the foundation soils as existing embankment has been in place since 1965 and added load to the existing embankment will be minor. Settlement should be reduced from the original settlement of 594 mm (1.95 ft) measured at settlement platform #5 where 12.80 m (42 ft) fill height was added and the original settlement period was 180 days. At the significant left side widen, additional fill is estimated at 7.62 m (25 ft) maximum height with an estimated maximum settlement of 305 mm (12 in). Houghs Method for settlement calculations is somewhat less at approximately 127 mm (5 in). The settlement period is estimated at approximately 180 days, however the actual settlement period will be determined by the project engineer on the basis of settlement data in the field.

At the Abutment 4 left side widen, additional fill is estimated at 6.7 m (22 ft) maximum height. Existing fill for Abutment 4 has been in place since 1965 so settlement should be reduced from the original settlement of 335 mm (1.1 ft) measured at settlement platform #6 where 10.06 m (33 ft) fill height was added and the original settlement period was 180 days. Our estimated maximum settlement is approximately 178 mm (7 in)). Houghs Method for settlement calculations is somewhat less at approximately 127 mm (5 in). OSF recommends a fill settlement period of up to 180 days for the left side widen; however, the actual settlement period will be determined by the project engineer on the basis of settlement data in the field. Any settlement due to the addition of the right side sliver fill widening should be negligible in the foundation soils as the added load to the existing embankment will be minor. Again, all fills can be placed in accordance with Section 19-6 of the Standard Specifications.

Structure approach slab type N(9D) will be incorporated within the proposed bridge widenings.

Plumb, 1.2 m (4 ft) diameter, Cast-in-Drilled-Hole (CIDH) piles can be used to support the widening at bridge abutments. Plumb, 2.1 m (7 ft) diameter drilled shafts will be used at the bents for the widenings as shown below. CIDH pile capacities were calculated using the Federal Highway Administration's Drilled Shaft Manual (Pub. No. FHWA-HI-88-042) published July 1988. Permanent casing is recommended to be placed into bedrock to facilitate construction of the drilled shafts, prevent caving of loose soils and gravel/cobble lenses into the pile borings, and seal off ground water from entering the pile borings. OSF feels that permanent steel casing can be emplaced at least near specified tip elevation using a vibratory hammer. In discussions between Mr. Ron Jones and the author (for nearby Sorrento Viaduct, Br. No. 57-0513R/L, March and April, 2000) the practice of drilling ahead of the casing before dropping the casing into place is considered undesirable as caving of loose soils and gravel/cobble lenses would create voids between the casing and surrounding soil, thus compromising the lateral capacity of the pile. However, drilling slightly ahead of casing in the basal gravel/cobble lenses and within bedrock will probably be necessary. OSF assumes no additional axial geotechnical capacity for permanent steel casing that will be installed to aid in construction of CIDH piles shown below.

Los Penasquitos Creek Bridge, Br. No. 57-0511 – right side sliver widen:

Support Location/ Type & Diameter	Design Loading			Nominal Resistance		Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
	Compression kN (tons)	Tension kN (tons)	Lateral kN (tons)	Compression kN (tons)	Tension kN (tons)					
Abut 1/CIDH 1.2 m (4 ft)	N/A			**2725 (305)**	**0**	3.66 (12.0)	+17.00 (+55.8)	-12.19 (-40.0)	-15.85(1) (-52.0)(1)	-15.85 (-52.0)
Bent 2/CIDH 2.1 m (7 ft)				**11600 (1300)**		7.62 (25.0)	+6.00 (+19.7)	-11.58 (-38.0)	-19.20(1) (-63.0)(1)	-19.20 (-63.0)
Bent 3/CIDH 2.1 m (7 ft)				**11600 (1300)**		9.75 (32.0)	+6.00 (+19.7)	-10.97 (-36.0)	-20.73(1) (-68.0)(1)	-20.73 (-68.0)
Abut 4/CIDH 1.2 m (4 ft)	N/A			**2225 (250)**	**0**	3.96 (13.0)	+15.00 (+49.2)	-12.19 (-40.0)	-16.15(1) (-53.0)(1)	-16.15 (-53.0)

Notes: Design tip elevation is controlled by the following demands:(1)Compression; (2)Tension; (3)Lateral Loads

Los Penasquitos Creek Bridge, Br. No. 57-0511 – left side widen:

Support Location/ Type & Diameter	Design Loading			Nominal Resistance		Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
	Compression kN (tons)	Tension kN (tons)	Lateral kN (tons)	Compression kN (tons)	Tension kN (tons)					
Abut 1/CIDH 1.2 m (4 ft)	N/A			**2725 (305)**	**0**	3.35 (11.0)	+17.00 (+55.8)	-13.11 (-43.0)	-16.46(1) (-54.0)(1)	-16.46 (-54.0)
Bent 2/CIDH 2.1 m (7 ft)				**11600 (1300)**		7.62 (25.0)	+6.00 (+19.7)	-13.11 (-43.0)	-20.73(1) (-68.0)(1)	-20.73 (-68.0)
Bent 3/CIDH 2.1 m (7 ft)				**11600 (1300)**		9.75 (32.0)	+6.00 (+19.7)	-13.41 (-44.0)	-23.16(1) (-76.0)(1)	-23.16 (-76.0)
Abut 4/CIDH 1.2 m (4 ft)	N/A			**2225 (250)**	**0**	3.96 (13.0)	+15.00 (+49.2)	-13.41 (-44.0)	-17.37(1) (-57.0)(1)	-17.37 (-57.0)

Notes: Design tip elevation is controlled by the following demands:(1)Compression; (2)Tension; (3)Lateral Loads

When pile nominal resistance in tension is provided by DSD (Division of Structures Design), OSF can then provide design pile tip elevations in tension. Also, if the pile tip elevation is controlled by lateral demands, the designer is responsible to present correct foundation data, governed by lateral control, on the foundation plans.

*Ground elevation at the bottom of Los Penasquitos Channel was surveyed in the area (based on NAVD 88 datum) at +7.19 m (+23.6 ft) on east side of the SBND Rte. 5 bridge (Los Penasquitos Creek Bridge, Br. No. 57-0511, just east of existing Bent 2). Pile cutoff elevations for supports within the Channel for the proposed widening of Br. No. 57-0511 are approximately at 1.22 m (4 ft) below this surveyed elevation. Proposed supports will be just downstream of the above surveyed elevation.

Axial compression values noted in the tables above are based on skin friction only within the rock. End bearing was not considered due to working below the water table and the possibility that cleaning out the bottom of pile borings effectively may be rather difficult at depth and may make it difficult to realize substantial end bearing using Caltrans standard pile vertical deflection criteria of 13 mm (0.5 in).

Bedrock topography (top of rock) is often quite variable across short lateral distances. Due to this fact, the pile data table above includes intended length of the rock socket at each support. The intended length of the rock socket should be measured from the bottom of the permanent casing down to the pile specified tip elevation. OSF feels that permanent casing should be seated into rock approximately 0.61 m (2 ft). If the bedrock slope is steeper than expected, the permanent casing may need to be seated slightly deeper to seal out water and potential caving soils.

Constructability

As mentioned above, OSF recommends installation of permanent casing to be placed into bedrock to prevent caving of loose soils and gravel/cobbles lenses into pile borings and help seal off ground water from entering the excavations once seated into rock. OSF and OGEE feel that using a vibratory hammer to place steel casing down to a level close to casing specified tip elevation would facilitate pile construction and effectively reduce creation of voids along the pile length by undesireable caving of loose soils and subrounded cobble/gravel material. Drilling ahead of the casing, especially within the upper loose/soft soil zones should be avoided to reduce caving and creation of voids, thus compromising lateral pile capacity. OSF anticipates center relief drilling to facilitate casing advancement. Hard slow drilling [through hard metavolcanic cobble zones (cobbles up to 200 mm diameter), cobble-size mudstone rock fragments, and bedrock] is anticipated during installation of permanent casing and CIDH piles (rock sockets). Drilling ahead of casing may be required, in order to advance casing within the middle and lower gravel/cobble lenses and within bedrock. Once casing is seated into bedrock, drilling for the rock sockets can be completed

At the nearby Rte. 5/805 Separation (Widen), Br. No. 57-0512, a Caliper log within Boring 99-2 (proposed Bent 5 – Right Side Widen) which was an uncased hole, shows that caving happens readily within shallow loose/soft often saturated alluvium and within the sand and gravel/cobble lenses overlying bedrock. This alluvial material located just upstream is very similar to shallow soils at the subject bridge (just downstream on Los Penasquitos Creek) which are expected to cave easily if left uncased.

Ground water should be anticipated at relatively shallow depths during CIDH pile construction. Static ground water was measured at corrected elevation +6.78 m (+22.2 ft) within Test Boring B-4C as shown on the As Built LOTB. Nearby Boring 99-1 for the Sorrento Viaduct (Br. No. 57-513L) shows static ground water measured at +7.77 m (+25.5 ft) elevation. Water flows intermittently within the channel also. The bottom of all excavations should be cleaned of loose debris before placing concrete.

Clay mineralogy within formational material appears sensitive to the introduction of fresh water, which could cause swelling of clays and slicking of borehole walls, resulting in reduced pile/soil skin friction capacity. OSF feels that a mud/polymer expert should be consulted and be available to the contractor to advise on proper drilling fluid/slurry chemistry in order to prevent clay swelling. OSF feels that seating permanent casing into the formational mudstones/claystones/siltstones should help seal off ground water from reacting with the formational clays.

At proposed Bent 2 for the right side sliver widen (Los Penasquitos Creek Bridge) and proposed Bent 13 for the S5/S805 Truck Connector, OSF recommends staggering time of construction of CIDH piles due to extreme closeness of CIDH pile supports. Even though permanent casing seated into the top of bedrock will be used for all pile construction in this area, some of the bedrock below the permanent casing may be saturated uncemented soil-like formational sands that may tend to cave.

Corrosiveness

Laboratory tests of one soil sample taken from Boring 99-2 [depths 19.81 to 20.27 m (65.0 to 66.5 ft)] show fill has a pH of 8.27. Laboratory tests of composite soil samples (taken within Boring 99-1 for nearby Retaining Wall No. 524) southeast of the bridge, indicate that fill and native material are corrosive. Corrosion tests on embankment fill show a pH of 7.48, minimum resistivity of 475 ohm-cm, sulfate and chloride content were measured at 5730 and 760 ppm, respectively. Corrosion tests on alluvial material show pH ranges from 7.48 to 7.98, minimum resistivity ranges from 475 to 746 ohm-cm, sulfate and chloride content were measured at 6000 to 360 ppm and 230 to 150 ppm, respectively. OSF feels that the Corrosion Technology Branch should be consulted regarding test results and possible recommendations.

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